HYDRAULIC DESIGN OF PERVIOUS CONCRETE
HIGHWAY SHOULDERS

A THESIS IN
Civil Engineering

Presented to the Faculty of the University
Of Missouri-Kansas City in partial fulfillment of
the requirements for the degree

MASTER OF SCIENCE

By
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B.S. University of Missouri-Kansas City, 2012

Kansas City, Missouri
2013
ABSTRACT

Stormwater drainage has been a factor in roadway design for years. Now stormwater quantity and quality are also becoming regulated for roadways. As regulations of stormwater management continue to increase so does the need for more viable and effective management practices. The research presented and discussed in this thesis presents the option of using pervious concrete in highway shoulders as a best management practice for stormwater management. Research focused on the hydraulic response of pervious concrete pavements exposed to sheet flowing water.

Pervious concrete samples were placed in a hydraulic flume to determine capture discharges, infiltration rates, and by-pass flowrates for a broad range of void contents, across a broad range of pavement cross slopes. The results demonstrate that the capture discharge and infiltration rates are inversely related to the cross slope of the pavement. Results also showed the infiltration rate of the permeable pavement exposed to sheet flowing water, in the model, is significantly lower than the measured infiltration rate.

Pervious concrete samples were also tested to determine hydraulic response when exposed to clogging associated with sand used in roadway de-icing. The results of the
clogging of the permeable pavements followed similar trends as the unclogged samples, with the only difference being a more significant reduction in infiltration rates at higher applications of sand. Preliminary discussion of a design methodology is included with a design example.
The faculty listed, appointed by the Dean of the School of Computing and Engineering, have examined a thesis titled “Hydraulic Design of Pervious Concrete Highway Shoulders,” presented by Nathan A. Grahl, candidate for the Master of Science Degree, and certify in their opinion it is worth of acceptance.

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The author would like to thank Dr. John Kevern and Dr. Jerry Richardson without whom none of the research in this thesis could have been accomplished.
CHAPTER 1
OVERVIEW

Stormwater management is now a ubiquitous concern for initial design, during construction, and for long-term site performance. Construction of impervious objects such as roadways and buildings prevents rainfall from infiltrating and decreases the time of concentration, increasing runoff volume. The runoff from impervious surfaces is then transported through storm drainage systems in either enclosed pipes or surface ditches and channels. Increased runoff volume is a significant cause of stream erosion, especially in urban areas. Impervious objects can increase the amount of storm water runoff by as much as 16 times (Clar 2004). The single biggest contributor to surface water impairment is the pollution carried by stormwater from impervious surfaces (Ferguson 2005). To help managed stormwater the Environmental Protection Agency (EPA) has guidance for Best Management Practices (BMPs) to manage the quality and quantity of stormwater runoff.

Permeable pavements are a unique stormwater BMP where a high permeability surface allows the stormwater to rapidly exfiltrate to an underlying holding area. Pervious concrete is a hydraulic cement mixture produced with little to no fine aggregate to help maintain a 15-30% water-permeable void ratio (ACI 2010). Large particles like metal vehicle pieces are caught on the surface while smaller soil particles pass through to the underlying aggregate layer to settle out. Microbes in the soil degrade the nutrients, oils, and greases to protect surface water (Obia 2007). The water volume is reduced through
absorption, evaporation, and infiltration. Any water discharged from drain tiles in the systems is delayed, minimizing the hydrologic peak, cooled, and filtered. Transitioning pervious concrete from parking areas to highway shoulders will allow stormwater management within right of way, which will keep sediment and pollutants out of highway drainage channels to extend the functional lifespan of the channel. The filtering feature of pervious concrete will help reduce the amount of debris entering highway drainage channels.

Managing highway stormwater runoff requires large amounts of infrastructure, from highway ditches to drain inlets and piping. Pervious concrete allows infiltration of stormwater into the ground with little to no conveyance. With pervious concrete shoulders, a significant amount of the infrastructure required for stormwater containment could be removed. Permeable highway shoulders would also enable stormwater to infiltrate into the ground at a slow rate compared to conventional drainage. The slower rate would help in erosion prevention by stabilizing the vegetation in roadside banks. The slower rate would provide support to the root system of vegetation in roadside banks. With a portion of stormwater penetrating directly into the soil, the amount of capacity needed in roadside channels could be decreased or maintained for more resilient capacity. This decrease in capacity equates to shallower ditches, which improve highway safety and roadside maintenance.

Three design considerations are required when using pervious concrete;

1) Mixture design/proportioning,

2) Hydrologic design, and
3) Hydraulic design.

Significant work has been performed on mixture proportioning and pervious concrete has been successful in harsh environmental conditions and heavy traffic loading (Kevern 2008a, Kevern 2005, Kevern 2008b, and Erikson 2006). Hydrologic design of pervious concrete is similar to stormwater detention ponds with programs and guidance available from many national sources (ACI 2009, Leming 2007). However, information regarding the hydraulic design of pervious concrete is non-existent. Care is given to the mixture proportioning, training is provided to the contractors and producers, the engineers design the detention and drawdown systems, but no consideration is provided for how the stormwater will actually be conveyed into the system. When pervious concrete is utilized in parking areas with little contributing run-on or sediment load, merely stating the pavement should be pervious may be acceptable. When pervious concrete is designed as a functional link in the stormwater management design, hydraulic aspects cannot be ignored.

The implementation of pervious concrete used in highway shoulders has already begun. Construction of pervious concrete shoulders for stormwater management was completed in the summer of 2012 by the Nevada Department of Transportation (NDOT), and in St. Louis Missouri by the Missouri DOT (MoDOT) (Kevern 2012). NDOT has also funded a pervious shoulder maintenance program to help address concerns which will arise from lack of hydraulic design guidance. The National Cooperative Highway Research Program (NCHRP) has also provided funding for the research of design, constructability, safety, and longevity of permeable pavements used in highway shoulders (25/25: Task 82).
Before permeable pavements become utilized more often for shoulders, this crucial area of hydraulic design must be addressed.

To present all necessary information and data completely and properly this thesis has been divided into seven significant chapters. Each chapter focuses on a different aspect of research or results, and may be sub divided into sections to further ease the understanding of the information provided.

Chapters one and two focus on a brief overview of the research performed and presented in this thesis. Chapters one and two also contain the background of the topics covered in this thesis.

Chapter three and four contain information regarding concrete and hydraulic testing, respectively. Chapter three contains the concrete mixture designs, the concrete testing, and the results of testing. Chapter four contains the model set-up and information regarding how the hydraulic components and data were determined and calculated.

Chapters five and six contain the results and discussion of the data for unclogged and clogged testing, respectively.

And Chapter seven contains explanation and example of the hydraulic design of pervious concrete pavements for use in highway shoulders.
CHAPTER 2
LITERATURE REVIEW AND INTRODUCTION

In 1948 the Environmental Protection Agency (EPA) enacted the Federal Water Pollution Control Act, this led the path the modern day Clean Water Act (EPA 2008). In the 1970s the Clean Water Act partnered with Indian tribes and focused primarily on the integrity of water quality through the chemical aspects. Initial regulation enforced by the Clean Water Act made it unlawful to discharge pollutant from point sources into navigable waters without permits. The EPA enacted the National Pollutant Discharge Elimination System (NPDES) as a regulatory permitting program to control the “point source” discharges. NPDES permits were primarily geared towards traditional point source facilities, such as municipal sewage treatment plants and industrial wastewater facilities (EPA 2008). It wasn’t until the late 1980s that focus on polluted runoff from nonpoint sources, such as runoff from streets, construction sites, and other “wet-weather” sources, was implemented.

Point source pollutants are pollutants from a known source such as wastewater treatment, where nonpoint source pollutants come from unknown sources such as agricultural infiltration and runoff. Over the last forty years the leading source of pollutants has shifted from point sources to nonpoint sources due largely to the Clean Water Acts regulatory programs controlling point sources of pollution (EPA 2008).
A part of the hydrologic cycle is infiltration of rain water into the ground (Bedient 2008). During this infiltration the rain water is allowed to slowly filter through the soil and rock, and then replenishes ground water aquifers. Stormwater that isn’t absorbed into the ground is classified as runoff. This runoff can be further classified into urban runoff and rural runoff. Initially urban and rural runoffs contain the same pollutants, such as sediment, nutrients, oxygen-demanding materials, and bacteria (GWQ 1997). Due to the growth in urban areas the amount of impervious area has grown, disabling the infiltration of stormwater into the ground. This results in an increased amount of surface water runoff, which eventually finds its way into surface bodies of water. Figure 1 shows the effects of development on surface water runoff.

Figure 1 Effects of development on stormwater runoff (EPA 2004).
While point source discharges into navigable bodies of water are highly regulated, it wasn’t until recently that the nonpoint discharges were regulated. These nonpoint source discharges from urban surface runoff carry large amounts of pollutants to surface water due to the fact that they aren’t filtered through the ground and collect more pollutants and debris from impervious urban areas. Congress identified urban runoff as a leading nonpoint source of pollution to surface waters (EPA 2005).

In the early 1990s the Clean Water Act switched from a program-by-program, source-by-source, pollutant-by-pollutant approach on pollution control to a more overall look based on watersheds (Introduction EPA). The watershed approach focuses on the waterbody protection and restoration instead of focusing on just pollution control. This new approach allows the EPA to address all water quality issues (EPA 2008).

Five basic principles guide the watershed approach adopted by the Clean Water Act: Place-based focus, stakeholder involvement and partnerships, Environmental goals and objectives, problem identification and prioritization, and Integration of actions (EPA 2005). Place-based focus allows the activities to be directed in specific geographic areas. The stakeholder involvement and partnerships incorporates a partnership among public and private groups to involve the people most affected by management decisions. Environmental goals and objectives focus the initiatives on improvements of the water resource rather than programmatic objectives. Problem identification and prioritization uses scientific data and methods to identify and prioritize threats to human and ecosystem health. And the integration of actions principle allows action to be taken in comprehensive and integrated manners (EPA 2005).
A section under the watershed approach includes a management model based on impervious cover (EPA 2005). This approach accounts for the current and future quality of streams and other water resources at the subwatershed scale (EPA 2005). The impervious cover management model utilizes subwatersheds as primary management units because: the influence of impervious cover is typically first evident on headwater streams, smaller scales can locate and identify impacts of individual sources of pollutants, subwatersheds are generally within the borders of one or two jurisdictions, and assessments and evaluations can be concluded quicker due to the smaller size of subwatersheds (EPA 2005).

The impervious cover model classifies subwatersheds as sensitive, degrading, or nonsupporting; based on percentage of impervious cover (EPA 2005). In 1983 the results of the EPA’s Nationwide Urban Runoff Program (NURP) were published. The NURP results showed evidence that watershed imperviousness directly affects the volumetric runoff of the site (EPA 1983). In addition to the effects on surface water urbanization has effects on multiple parts of the water cycle, as shown in Figure 2 (EPA 2005).
Due to urbanization a significant portion of the water cycle is being transferred into surface water runoff as seen in Figure 2. This water is transferred into stormwater conveyance systems before discharge into surface water bodies.

Highway Drainage Design

Stormwater has been an issue ever since roadways have been around. In the early days when the majority of roads were unpaved, dirt roadways, one substantial storm event could leave the dirt roads un navigable. These primitive roads made interstate travel a tedious and time consuming process. Since the 19th century roadways were recognized as
state and local responsibilities, and since privately owned railroads were the primary source of interstate commerce roadways were the least of state’s concerns (Weingroff 1996).

With the introduction of the Model T, by Henry Ford, in 1908, and the popularity of bicycles growing the condition of roadways started to become a concern. Farmers wanted all-weather roads so they could get their products to market, and the automobile industry wanted hard surfaced interstate roads (Weingroff 1996, 1). With the pressure from farms, automobile industries, and other activist groups, congress began to investigate options for federal funding of interstate roadways. After lengthy investigations and legislative options congress approved the Federal Aid Road Act, and President Wilson signed the bill in 1916 (Weingroff 1996, 1). Even with the federal funding for roadways under the Federal Aid Road Act in 1916, dirt and muddy roadways were still common in the 1930s (Weingroff 1996, 3).

Several legislations were passed and organizations were formed but it wasn’t until the passing of the Federal-Aid Highway Act of 1956 that uniform interstate design standards were set in place (Weingroff 1996, 2). The standards set in the 1956 Federal-Aid Highway Act accommodated for traffic forecasts for 1975 (Weingroff 1996, 2). Ten years later the Federal Highway Administration (FHWA) was created. The Department of Transportation Act of 1966 changed the name of the Bureau of Public Roads to the Federal Highway Administration (FHWA) (Weingroff 1993). The primary goal of the FHWA was the completion of the interstate system, but there are several other responsibilities of the FHWA (Weingroff 1993).
The FHWA has published numerous documents detailing highway design guidelines, including hydraulic design of highways. In highway hydraulics there are two components that must be accounted for which are hydrology and hydraulics (FHWA 2008). The hydrologic aspect accounts for the quantity and frequency of the runoff, surface water, and ground water (FHWA 2008). The hydraulic aspect accounts for the design of structures to collect and transport storm water off and away from highway right-of-ways (FHWA 2008). There are two categories for highway drainage facilities, open channel or closed conduit facilities (FHWA 2008). Open channel of closed conduit facilities can be used individually or together, Figure 3 shows an example of the two systems working together. In figure 3 the roadway channel and median swale represent open channel drainage facilities, while the median inlet and culvert represent closed conduit facilities.

Figure 3 Highway drainage facilities (Hydraulic 2008).
FHWA’s Hydraulic Design Series (HDS) No. 4 states the primary purpose of highway drainage facilities is to “prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway (FHWA 2008).” In order to fully capture and transport stormwater from roadways, calculation to determine flow rates into the system must be performed. The first step in calculating stormwater flow rates is to determine the appropriate flood frequency. After this flood frequency is verified to be acceptable for design, the flow need to be captured is calculated using the time of concentration, rainfall intensity, and storm duration (ACI 2010).

The time of concentration is the amount of time that it takes for water from the storm entering the watershed is equal to the discharge leaving the watershed, entering the storm sewer system (FHWA 2008).

The calculation of the quantity of water the facility must convey may at first seem tedious and labor intensive, but there are proven and sound design procedures that are suitable for more day-to-day design problems (FHWA 2008).

In every storm there is a peak discharge, or a max discharge, this is the discharge generally used in design calculations. Figure 4 shows a typical hydrograph for a storm event, indicating the peak flow, and the time to peak flow. The peak rate of runoff can be calculated using the drainage area and average rainfall intensity, assuming a uniform rate of intensity (FHWA 2008).
After calculating the amount of storm runoff needed for capture, the design can be focused on the hydraulics, or the system used to capture and convey the stormwater. Although a majority of rural highways use open air drainage channels to convey storm water away from the site, some highways require close-conduit drainage facilities.

The drainage channels used need to be designed to adequately convey the design discharge capacity, and are usually lined with vegetation, with rock or paved linings being utilized where vegetation will not control erosion (AASHTO 2011). Fill slopes are used to direct storm water from the highway shoulder into drainage channels. When the fill slopes are subjected to erosion concerns due to the runoff from the roadway, closed conduit systems are implemented (AASHTO 2011).

Closed conduit systems include the use of dikes, inlets, and chutes of flumes, and are more suitable for urban highway applications (AASHTO 2011). Typical closed conduit systems utilize curb and gutter to transport water from the roadway surface to drainage.
inlets. Drainage inlets on highways should be located appropriately to prevent the spread of water from entering more than a reasonable distance into the traveled way (AASHTO 2011). Due to the amount of traffic and the speeds of traffic on highways, the allowable spread width is significantly smaller than on residential roadways (FHWA 2008). Drainage inlets generally operate under weir flow conditions and are designed based on type, location (on grade or in a sag), the gutter design, and to operate while partially clogged (FHWA 2008).

In addition to stormwater conveyance systems, the peak flow can be dramatically reduced by providing detention basins. Detention basins are especially beneficial when designing highway drainage where the existing downstream drainage facilities are inadequate (AASHTO 2011).

Pervious Concrete as a BMP

As discussed earlier there is a strong correlation between landscape perviousness and surface water runoff. With that theory an ideal option would be to increase surface perviousness to reduce runoff on highways. The sustainability of pervious concrete has transformed it into a viable Best Management Practice (BMP). Pervious concrete has been around for hundreds of years, but was not brought to the United States until after World War II (CPG 2013).

Pervious concrete is a specialty concrete where the amount of fine aggregate is either significantly reduced or eliminated, to allow water to intentionally pass through the surface of the pavement and through the voids (CPG2013). Pervious concrete contains and
filters stormwater making it an ideal on site BMP for stormwater management. If this stormwater can be treated on site through the pervious concrete, the need of conveying stormwater to water treatment plants is eliminated, and in return eliminates the need to treat this stormwater. When pervious concrete is used in regions with large amounts of clay soils, the pervious concrete is placed on a larger rock base to create a stormwater system (CPG 2013). The use of the rock base and pervious concrete over clay soils allows for a detention area (in the rock base) for stormwater. These systems have been shown to lose 0.75 inches of water a day to evaporation, this means that a three inch storm event will evaporate in four days (CPG 2013). Hydrological factors to consider when determining pervious concrete affects for stormwater are: how much water needs to be mitigated, how much water will the rock base hold, and how thick of a base should be used (generally 12 inches) (CPG 2013). If the pervious concrete structure can’t mitigate the complete amount of flow associated with the design flood frequency, overflow pipes can be utilized. These overflow pipes can connect the concrete to other BMPs such as bioswales or to conventional stormwater sewer systems (CPG 2013). The use of pervious concrete over other conventional BMPs’ is optimal due to its help in achieving Low Impact Developments, where you build up, keeping everything onsite, as opposed to building out (CPG 2013).

Other benefits in addition to water detention from pervious concrete are: water filtration, pervious concrete improves the health of trees, recharges ground water aquifers, reduces pavement effects on urban heat island, and contributes to site LEED accreditation (Carlson 2007).
Like any new hot topic in construction, pervious concrete had to prove itself. Pervious concrete in the United States began in Florida and other southern coastal states and slowly migrated to other states with different acceptance and rejection (CPG 2013). With pervious concrete gaining acceptance in the coastal communities, there was hesitation for it to migrate to use in the Midwest. This hesitation was largely due to the freeze and thaw susceptibility of pervious concrete and it’s abilities to withstand the harsher climates (CPG 2013). The voids in pervious concrete allow water to freely flow through it, therefore preventing the concrete from breaking apart due to the expansion of water when freezing. This resistance to breaking up during freeze thaw cycles makes pervious concrete more desirable than conventional concrete (CPG 2013). This system can be seen in Figure 5, below.

Figure 5 : Typical pervious concrete pavement system (CPG 2013).
There are four main parts to the successful implementation of pervious concrete as a stormwater BMP, which are: the design of the pavement, the mixture design delivered by the ready mix producer, the contractor placing the pervious concrete, and the owner with regards to maintenance of the pavement (CPG 2013). Traditional concrete has been put through all of the tests and therefore contractors and producers know how to design handle and construct with traditional concrete. But pervious on the other hand has not been put through as many tests and contractors and producers are not as familiar with it. This unfamiliarity leads to numerous problems with the quality of pervious concrete both in the mixture and in the placement. Special care should be taken when placing pervious concrete.

The low water content and higher void content of pervious concrete make construction with it more difficult. With pervious concrete there is a correlation between strength and permeability, as the permeability decreases the strength increases, and vice-versa. This correlation requires stricter control of the mixture proportioning. The low water content of pervious concrete also requires transportation and placing times to be lower, to prevent the concrete from setting before it’s in place. Pervious concrete also requires special consideration when placing it. Pervious concrete is not pumpable which makes the logistics of placing it more difficult. Placement of pervious concrete should be continuous and have rapid spreading, as well as rapid strikeoff (Kevern 2008). Finishing is not required with pervious concrete; once it is compacted it is finished and can be cured.

If the proper care and attention are put into the design, production, and maintenance of the pervious concrete pavement are taken, it can be a very successful stormwater BMP.
As familiarity and knowledge of pervious concrete continues to grow, so will the use and implementation of pervious concrete for sustainability.
CHAPTER 3
PERVIOUS CONCRETE DESIGN AND TESTING

Materials
The materials used in the pervious concrete mixture were locally available, standard, materials. The coarse aggregate was an ASTM C33 size #8 crushed limestone with a dry-rodded void content of 40% (ASTM 2011). A polycarboxylate high-range water reducing agent was used at 4 oz/cwt (2.5 mL/kg). The unit cwt associated with the high-range water reducing agent stands for century weight, the 4oz/cwt indicates that there was 4 ounces per 100 pounds of cementitious material in the mixture.

Mixture Design
For testing to fully analyze pervious concrete, a broad range a void contents were tested. To achieve this goal a broad range of concrete mixtures were designed by void content. Several mixtures were placed and analyzed, and three of those mixtures were selected for further testing. The initial mixture utilized different compaction techniques to achieve various void contents. The void contents achieved from this mixture are: 20%, 35%, and 45%. The mixture proportions for the 20%, 35%, and 45%, samples are shown in Table 1.
Table 1 Pervious Concrete Mixture for the 20%, 35%, and 45% Void Samples.

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>lb/yd³</th>
<th>kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Aggregate</td>
<td>2620</td>
<td>1555</td>
</tr>
<tr>
<td>Portland Cement</td>
<td>550</td>
<td>325</td>
</tr>
<tr>
<td>Water</td>
<td>160</td>
<td>95</td>
</tr>
</tbody>
</table>

To fill out the range of void contents mixture designs utilizing higher paste contents were investigated. The additional amount of paste created lower voids. Several mixtures were placed utilizing different amounts of paste, with varying results. Two of these mixtures were produce void contents of 15% and 25% and were chosen for further investigation. The mixture designs for sample void contents 15% and 25% are shown in Tables 2 and 3, respectively.

Table 2 Pervious Concrete Mixture for the 15% Void Sample.

<table>
<thead>
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<th>Ingredient</th>
<th>lb/yd³</th>
<th>kg/m³</th>
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</thead>
<tbody>
<tr>
<td>Coarse Aggregate</td>
<td>2450</td>
<td>1110</td>
</tr>
<tr>
<td>Portland Cement</td>
<td>650</td>
<td>295</td>
</tr>
<tr>
<td>Water</td>
<td>195</td>
<td>90</td>
</tr>
</tbody>
</table>
Table 3 Pervious Concrete Mixture for the 25% Void Sample.

<table>
<thead>
<tr>
<th></th>
<th>lb/yd$^3$</th>
<th>kg/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Aggregate</td>
<td>2530</td>
<td>1150</td>
</tr>
<tr>
<td>Portland Cement</td>
<td>600</td>
<td>270</td>
</tr>
<tr>
<td>Water</td>
<td>180</td>
<td>80</td>
</tr>
</tbody>
</table>

Several of the mixture designs utilizing additional amounts of paste were too fluid and not pervious enough for testing. With higher paste pervious concrete designs there is a chance that the paste will separate from the mixture. This occurred with several mixtures tested and an example is shown in Figure 6. In Figure 6 it is apparent that a majority of the paste settled to the bottom of the sample.
Samples

Nine cylinders and three test blocks were placed of each sample type, for testing of the concrete properties in triplicate. Mixing and curing of the concrete samples was performed in accordance with ASTM C192 (ASTM 2007). The blocks were 14 in. by 14 in. by 3 in. (355.6 mm x 355.6 mm x 76.2 mm) to allow infiltration testing per ASTM C1701 (ASTM 2009). An example of these blocks is shown in Figure 7. As shown in Figure 7, the sides are the samples were sealed with duct tape to ensure the water infiltrated through the sample.
Cylinder samples were 4 in. (101.6 mm) diameter by 8 in (203.2 mm) tall. The 20%, 35%, 45% void content samples were placed using the same mixture design with varied amounts of compaction (Kevern 2010). The 45% void sample had no compaction, the 35% void sample was compacted in two lifts with three jigs per lift, and the 20% void sample was placed in three lifts with 10 jigs per lift. The 15% and 25% void content samples were compacted in two lifts with three jigs per lift. The jigging procedure is defined in ASTM C29, the cylinder is filled a third of the way at a time and is allowed to drop two inches against a firm base for the designated number of jigs (ASTM 2003). The data will be presented by mixture, with the 20%, 35%, and 45% samples grouped together,
and then the 25% and 15% samples. Fresh unit weight was measured on the cylinders and the appropriate amount massed for the slab samples to ensure equal voids.

Void Content

Hardened void content and unit weight was determined according to ASTM C1754 (ASTM C1754 2012). The basic principles and calculations in ASTM C1754 were modified to allow for the testing of void content on the pervious concrete blocks. Hardened void content is determined by weighing the samples submerged in water and then dried. The modifications to ASTM C1754 included modified techniques to obtain the submerged weight of the concrete blocks. These modifications are shown in Figure 8. The percentages used for sample identification are the hardened void content of the blocks. The hardened void content of the cylinders is shown in Table 4.
Figure 8 Techniques for testing voids of pervious concrete.

Table 4 Cylinder Void Testing Results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cylinder Voids</th>
<th>Std. Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>47%</td>
<td>0.226%</td>
</tr>
<tr>
<td>35%</td>
<td>38%</td>
<td>0.657%</td>
</tr>
<tr>
<td>25%</td>
<td>31%</td>
<td>0.670%</td>
</tr>
<tr>
<td>20%</td>
<td>25%</td>
<td>0.404%</td>
</tr>
<tr>
<td>15%</td>
<td>23%</td>
<td>0.285%</td>
</tr>
</tbody>
</table>

Compressive Strength

Compressive strength was determined at 7 days on sulfur-capped specimens according to ASTM C39 (ASTM C39 2012). The results of the compressive strength testing are shown in Table 5.
Table 5 Compressive Strength Testing Results

<table>
<thead>
<tr>
<th>Sample</th>
<th>7 Day Compressive Strength (psi)</th>
<th>Std. Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>511</td>
<td>41</td>
</tr>
<tr>
<td>35%</td>
<td>1070</td>
<td>44</td>
</tr>
<tr>
<td>20%</td>
<td>1255</td>
<td>29</td>
</tr>
<tr>
<td>25%</td>
<td>2920</td>
<td>47</td>
</tr>
<tr>
<td>15%</td>
<td>3160</td>
<td>36</td>
</tr>
</tbody>
</table>

Infiltration

Infiltration of the samples was tested on the blocks in accordance with ASTM C1701 (ASTM 2009). ASTM C1701 is performed by determining the time a specified amount of water takes to infiltrate the sample. The water is poured onto the sample in a ring with a predetermined area, from the area of the ring and the time the water takes to infiltrate the sample, the infiltration rate can be calculated. The procedure for infiltration testing is considered a constant head test, were a predetermined weight of water is introduced into the samples at a constant rate. This procedure is shown in Figure 9, and the results are shown in Table 6.
Figure 9 Infiltration testing set-up.

In Figure 9 water is poured into the ring of PVC, and is kept at a determined height to sustain the constant head, and the time the water takes to penetrate the sample is used to determine the infiltration in inches per hour.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Infiltration (in/hr)</th>
<th>Std. Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>3944</td>
<td>244</td>
</tr>
<tr>
<td>35%</td>
<td>1784</td>
<td>65</td>
</tr>
<tr>
<td>20%</td>
<td>461</td>
<td>35</td>
</tr>
<tr>
<td>25%</td>
<td>781</td>
<td>177</td>
</tr>
<tr>
<td>15%</td>
<td>304</td>
<td>98</td>
</tr>
</tbody>
</table>
Mean Texture Depth
A sand patch test is used to determine the surface texture of concrete pavements, and was performed in accordance with ASTM E965 (ASTM E965 2006). The procedure to determine surface texture of concrete pavements includes spreading, in a circular pattern, a pre-determined volume of standard sand. From the diameter of the circle the surface texture can be calculated using the equation below.

\[ MTD = \frac{4V}{\pi D^2}, \]

Where,

MTD = Mean Texture Depth, in (mm),

V = Sample volume of sand, in³ (mm³),

And,

D = Average diameter of the area covered by standard sand.

Figure 10 shows the sand before spreading and after spreading. And Figure 11 shows the difference, in spread, between a high void sample (on the left) and a low void sample (on the right). Figure 11 depicts that the higher void sample has a smaller spread, due to the sand penetrating through the increased amount of voids, while, the lower void sample indicates a larger spread, due to lack of voids.
Figure 10 Sand patch test.

Figure 11 Sand patch test of high void (left) and low void (right) pervious concrete samples.
The calculated MTDs of the sand patch test are shown in Figure 12.

![Figure 12 Maximum texture depth calculated from sand patch test.](image)

### Summarized Testing Results

Table 7 shows a summary of all the determined concrete properties.

#### Table 7 Summarized Concrete Properties

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cylinder Void, %</th>
<th>7 Day Compressive Strength, psi</th>
<th>ASTM C1701 Infiltration (in/hr)</th>
<th>Mean Texture Depth, in</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>0.47</td>
<td>511</td>
<td>3950</td>
<td>0.30</td>
</tr>
<tr>
<td>35%</td>
<td>0.38</td>
<td>1070</td>
<td>1790</td>
<td>0.11</td>
</tr>
<tr>
<td>20%</td>
<td>0.31</td>
<td>1255</td>
<td>460</td>
<td>0.06</td>
</tr>
<tr>
<td>25%</td>
<td>0.25</td>
<td>2920</td>
<td>780</td>
<td>0.10</td>
</tr>
<tr>
<td>15%</td>
<td>0.23</td>
<td>3160</td>
<td>300</td>
<td>0.06</td>
</tr>
</tbody>
</table>
Figures 13, 14, and 15 show the compressive strength, infiltration rate, and mean texture depth verse the sample void content, respectively. The figures showing the infiltration rate and the mean texture depth show the results as expected. The spike in data shown in Figure 13 is due to the different mixture design used to obtain the range of voids.

Figure 13 Compressive strength verse sample void content.
Figure 14 ASTM C1701 infiltration rate verse sample void content.

Figure 15 Mean texture depth verse sample void content.
CHAPTER 4
HYDRAULIC TESTING

A hydraulic flume was designed and constructed to analyze hydraulic flow across the samples at different slopes and flow rates. The flume used for testing is shown in Figure 16. The flow rate was verified in triplicate. An in-line Venturi meter to measure the total flow rate ($Q_{\text{T}O\text{TA}L}$) into the system and two V-notch Weirs were used to measure the captured discharge (water flowing through the sample, $Q_{\text{INF}I\text{L}T\text{R}A\text{T}I\text{O}N}$), and the by-pass discharge (water flowing over the sample, $Q_{\text{BY-PASS}}$). Figure 17 shows an in-line view of the flume.

Figure 16 Hydraulic flume set up with flowrate indication.
The pervious concrete samples were placed in the hydraulic flume and sealed around the edges to prevent water from infiltrating through the edges between the concrete and the model. Standard plumbers putty was used to seal the edges. Figure 18 shows the pervious concrete block in the flume. Also in Figure 18 are four screws located in the corners of the sample. These screws were used for installation and removal of the sample.

The V-notch weirs were constructed from a piece of an eighth inch (3.175 mm) aluminum, and the notches were cut at a 22.5 degree angle, shown in Figure 19.
Figure 18 Pervious concrete sample in the hydraulic flume.

Figure 19 V-Notch weirs for calculating capture discharge (right) and by-pass discharge (left).
The use of v-notch weirs to determine flowrate requires the depth of water upstream of the weir. The depth of water upstream of the weir was measured in dead zones of the tailboxes to minimize the inconstancies associated with turbulent flow. This is shown in Figure 20, where a point gage was used in the infiltration tailbox to determine depth of water. The infiltration and by-pass flowrate are calculated using the following equation:

\[
Q = C_d \sqrt{2gh^{5/2}}; \quad \text{(eq. 1)}
\]

Where,

\[Q = \text{Flowrate, cfs,}\]

\[C_d = \text{Weir Coefficient,}\]

\[g = \text{gravity, 32.2 ft/s}^2\]

and

\[H = \text{Height of water upstream of the weir, in.}\]
The weir coefficient used in the equation above is used to calibrate the weir and is calculated for each individual weir. To calibrate the v-notch weirs a range of discharges, calculated from the calibrated venture meter, were analyzed and depths of water were noted. From there the weir coefficient, $C_d$, was back calculated. The calculated $C_d$ for the infiltration and by-pass v-notch weirs were 0.07 and 0.065, respectively.

The venture meter located upstream of the flume was used to calculate the total flowrate entering the system. Figure 21 shows the venturi meter used, and the equation for calculating flowrate is:

$$Q = C_d \sqrt{2gH};$$
Where,

\[ Q = \text{flowrate, cfs}, \]
\[ C_d = \text{Weir coefficient}, \]
\[ g = \text{Gravity, } 32.2 \text{ ft/s}^2, \]

and,

\[ H = \text{Height Difference, ft of water}. \]

Where,

\[ H = H_{IF}(SG_{Merrium} - 1) \]
\[ H_{IF} = \text{Height difference of dye in the venture, ft}, \]
\[ SG_{Merrium} = \text{Specific gravity of merrium dye, 2.95} \]

Figure 21 Venturi meter used for calculating flowrate into the system.
Highways and roadways have various slopes, to analyze field applications of the pervious concrete the slopes of different roadways needed to be mimicked. A hydraulic jack was used to vary the slope of the flume, and is shown in Figure 22. A full range of slopes were chosen for testing and they were: 0%, 0.5%, 1.0%, 2.0%, 5.0%, and 10.0%.

Figure 22 Hydraulic jack used to alter the slope of the flume.

The flume was designed to maintain a constant sheet flow of water through the flume. This water was flowing at subcritical velocities and is shown in Figure 23.
Figure 23 Steady flow of Water at Subcritical Velocities.
CHAPTER 5
RESULTS AND DISCUSSION (UNCLOGGED)

Maximum Capture Flowrate

The results presented in this chapter are for the hydraulic response of unclogged permeable pavements. Each sample was tested at six different slopes, across a range of flow rates. Figure 24 shows a performance curve for the high void samples at a 0% cross slope. The performance curve shown in Figure 24 is typical for all the samples. As the flow rate increases so does infiltration until by-pass occurs. A small increase in observed infiltration occurs as the flow depth increases until the pavement reaches a point of maximum consistent infiltration. After the point of consistency is reached the infiltration rate becomes independent flow rate.
Figure 24 Explanation of a typical performance curve

A performance curve was plotted for each block, void content, and slope, for a total of ninety performance curves. All plots were similar in nature and the plot shown at each void content for a 0% cross slope in Figures 25 through 29. The point of by-pass shown on Figure 24, was selected as the maximum capture flowrate for design calculations. That is the maximum amount of flow that will infiltrate the sample before overflow occurs.

The performance curves shown in Figures 25 through 29 indicate that as the void content is reduced, the amount of infiltration is also reduced, as expected. The performance curves also show the tendency of the amount of infiltration levels off, and reach a maximum infiltration flowrate, or Maximum Capture Flowrate. It is important to note that
at higher void contents the infiltration gradually increases before reaching MCF, while at the lower void contents the infiltration reaches MCF immediately, as seen in Figures 27, 28, and 29. While the main focus of this thesis is the flowrate at which the by-pass begins, the MCF’s of the specimens are important and Table 8 shows the MCF for the samples. The information in Table 8 is an average of the three blocks for each slope and void content. Figure 30 is a plot of the data represented in Table 8.

![Graph](image)

Figure 25 Performance curve for a 45% block with a 0% cross slope.
Figure 26 Performance curve for a 35% block with a 0% cross slope.

Figure 27 Performance curve for a 25% block with a 0% cross slope.
Figure 28 Performance curve for a 20% block with a 0% cross slope.

Figure 29 Performance curve for a 15% block with a 0% cross slope.
Table 8 Maximum Capture Flowrates

<table>
<thead>
<tr>
<th>Void Content</th>
<th>Pavement Cross Slope 0%</th>
<th>Pavement Cross Slope 0.5%</th>
<th>Pavement Cross Slope 1%</th>
<th>Pavement Cross Slope 2%</th>
<th>Pavement Cross Slope 5%</th>
<th>Pavement Cross Slope 10%</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>0.108</td>
<td>0.109</td>
<td>0.110</td>
<td>0.109</td>
<td>0.107</td>
<td>0.104</td>
</tr>
<tr>
<td>35%</td>
<td>0.053</td>
<td>0.052</td>
<td>0.052</td>
<td>0.051</td>
<td>0.049</td>
<td>0.048</td>
</tr>
<tr>
<td>25%</td>
<td>0.022</td>
<td>0.021</td>
<td>0.021</td>
<td>0.020</td>
<td>0.020</td>
<td>0.020</td>
</tr>
<tr>
<td>20%</td>
<td>0.013</td>
<td>0.012</td>
<td>0.011</td>
<td>0.011</td>
<td>0.011</td>
<td>0.010</td>
</tr>
<tr>
<td>15%</td>
<td>0.007</td>
<td>0.007</td>
<td>0.006</td>
<td>0.006</td>
<td>0.005</td>
<td>0.005</td>
</tr>
</tbody>
</table>

*Values are in cfs

Figure 30 Plot of pavement cross slope verse flowrate for the range of voids.

The data represented in Table 8 verifies that for any slope the MCF will significantly reduce as the void content is reduced. The data in Table 8 also shows that as the pavement cross slope is increased for any of the void contents the MCF does not have a significant change.
The performance curves were used to calculate a Maximum Capture Flowrate before ByPass (MCFPB) for each concrete block at each cross slope. To determine the MCFBP a linear trendline of the by-pass discharge line, seen in Figure 24, was used. The equation of the trendline of the by-pass discharge line was used to back calculate the point where by-pass began (where the trendline intercepts the x-axis), and this point is the MCFBP. The MCFBPs calculated for each respective void content are shown in Tables 9 through 13.

Table 9 MCFBP for 45% Void

<table>
<thead>
<tr>
<th>Slope</th>
<th>Block 1 (cfs)</th>
<th>Block 2 (cfs)</th>
<th>Block 3 (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.089</td>
<td>0.120</td>
<td>0.107</td>
</tr>
<tr>
<td>0.5%</td>
<td>0.092</td>
<td>0.111</td>
<td>0.095</td>
</tr>
<tr>
<td>1%</td>
<td>0.089</td>
<td>0.104</td>
<td>0.090</td>
</tr>
<tr>
<td>2%</td>
<td>0.083</td>
<td>0.104</td>
<td>0.093</td>
</tr>
<tr>
<td>5%</td>
<td>0.073</td>
<td>0.095</td>
<td>0.092</td>
</tr>
<tr>
<td>10%</td>
<td>0.072</td>
<td>0.086</td>
<td>0.084</td>
</tr>
</tbody>
</table>

*Values are in cfs

Table 10 MCFBP for 35% Void Content

<table>
<thead>
<tr>
<th>Slope</th>
<th>Block 1 (cfs)</th>
<th>Block 2 (cfs)</th>
<th>Block 3 (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.042</td>
<td>0.059</td>
<td>0.051</td>
</tr>
<tr>
<td>0.5%</td>
<td>0.037</td>
<td>0.056</td>
<td>0.048</td>
</tr>
<tr>
<td>1%</td>
<td>0.043</td>
<td>0.050</td>
<td>0.052</td>
</tr>
<tr>
<td>2%</td>
<td>0.034</td>
<td>0.049</td>
<td>0.045</td>
</tr>
<tr>
<td>5%</td>
<td>0.026</td>
<td>0.045</td>
<td>0.046</td>
</tr>
<tr>
<td>10%</td>
<td>0.022</td>
<td>0.038</td>
<td>0.040</td>
</tr>
</tbody>
</table>

*Values are in cfs
Table 11 MCFBP for 25% Void Content

<table>
<thead>
<tr>
<th>Slope</th>
<th>Block (cfs)</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.024</td>
<td>0.026</td>
<td>0.024</td>
<td></td>
</tr>
<tr>
<td>0.5%</td>
<td>0.019</td>
<td>0.022</td>
<td>0.020</td>
<td></td>
</tr>
<tr>
<td>1%</td>
<td>0.018</td>
<td>0.021</td>
<td>0.016</td>
<td></td>
</tr>
<tr>
<td>2%</td>
<td>0.018</td>
<td>0.023</td>
<td>0.016</td>
<td></td>
</tr>
<tr>
<td>5%</td>
<td>0.018</td>
<td>0.022</td>
<td>0.016</td>
<td></td>
</tr>
<tr>
<td>10%</td>
<td>0.016</td>
<td>0.023</td>
<td>0.012</td>
<td></td>
</tr>
</tbody>
</table>

*Values are in cfs

Table 12 MCFBP for 20% Void Content

<table>
<thead>
<tr>
<th>Slope</th>
<th>Block (cfs)</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>0.016</td>
<td>0.016</td>
<td>0.018</td>
<td></td>
</tr>
<tr>
<td>0.5%</td>
<td>0.014</td>
<td>0.011</td>
<td>0.013</td>
<td></td>
</tr>
<tr>
<td>1%</td>
<td>0.011</td>
<td>0.013</td>
<td>0.012</td>
<td></td>
</tr>
<tr>
<td>2%</td>
<td>0.012</td>
<td>0.010</td>
<td>0.014</td>
<td></td>
</tr>
<tr>
<td>5%</td>
<td>0.011</td>
<td>0.011</td>
<td>0.010</td>
<td></td>
</tr>
<tr>
<td>10%</td>
<td>0.005</td>
<td>0.009</td>
<td>0.008</td>
<td></td>
</tr>
</tbody>
</table>

*Values are in cfs
Table 13 MCFBP for 15% Void Content

<table>
<thead>
<tr>
<th>Slope</th>
<th>Block (cfs)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>0%</td>
<td>0.010</td>
<td>0.009</td>
<td>0.014</td>
</tr>
<tr>
<td>0.5%</td>
<td>0.007</td>
<td>0.007</td>
<td>0.010</td>
</tr>
<tr>
<td>1%</td>
<td>0.007</td>
<td>0.006</td>
<td>0.010</td>
</tr>
<tr>
<td>2%</td>
<td>0.007</td>
<td>0.006</td>
<td>0.007</td>
</tr>
<tr>
<td>5%</td>
<td>0.006</td>
<td>0.005</td>
<td>0.010</td>
</tr>
<tr>
<td>10%</td>
<td>0.007</td>
<td>0.004</td>
<td>0.006</td>
</tr>
</tbody>
</table>

*Values are in cfs

The average MCFBP of the three blocks was calculated for each void content, at each slope, to produce figures that represent the relationship between slope and MCFBP. Figures 31 and 32 show the relationship between slope and MCFBP. Figure 31 shows the samples that are from the same mixture with different compactions, and Figure 32 shows the samples that are from different mixtures.
Figure 31 MCFBP for the 45%, 35%, and 20% voids samples

Figure 32 MCFBP for the 25% and 15% voids samples.
The data displayed in Figures 31 and 32 show trends that indicate as the cross slope of the pervious concrete blocks increases the MCF decreases.

**Effective Intensity**

To provide a standard that could be used for design of pervious concrete highway shoulders the MCFBP from the model was converted to an effective intensity. The effective intensity is the maximum flowrate the pervious concrete can handle per unit length per unit width. To convert the model MCFBP into an effective intensity the equation below was used.

\[
I_E = \left( \frac{MCFBP}{L_M W_M} \right);
\]

Where,

- \(I_E\) = Effective Intensity, cfs,
- MCFBP = Mean Capture Flowrate before By-Pass, cfs,
- \(L_M\) = Length of the section used in the model, ft,
- \(W_M\) = Width of the Concrete block used in the model, ft.

The calculated effective intensities are shown in Tables 14. The behavior of the effective intensities mirrors the behavior of the MCFBP, due to the direct relationship.
Table 14 Effective Intensities for Varied Slopes

<table>
<thead>
<tr>
<th>Slope (Voids)</th>
<th>45% Voids</th>
<th>35% Voids</th>
<th>25% Voids</th>
<th>20% Voids</th>
<th>15% Voids</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0%</td>
<td>0.0762</td>
<td>0.037</td>
<td>0.014</td>
<td>0.012</td>
<td>0.008</td>
</tr>
<tr>
<td>0.5%</td>
<td>0.0718</td>
<td>0.034</td>
<td>0.013</td>
<td>0.009</td>
<td>0.006</td>
</tr>
<tr>
<td>1.0%</td>
<td>0.0684</td>
<td>0.035</td>
<td>0.012</td>
<td>0.009</td>
<td>0.006</td>
</tr>
<tr>
<td>2.0%</td>
<td>0.0678</td>
<td>0.031</td>
<td>0.012</td>
<td>0.009</td>
<td>0.005</td>
</tr>
<tr>
<td>5.0%</td>
<td>0.0627</td>
<td>0.028</td>
<td>0.009</td>
<td>0.008</td>
<td>0.005</td>
</tr>
<tr>
<td>10.0%</td>
<td>0.0584</td>
<td>0.024</td>
<td>0.009</td>
<td>0.005</td>
<td>0.004</td>
</tr>
</tbody>
</table>

Infiltration

Water flows across pavements in sheet flow, which is supercritical flow. Supercritical flow has a higher velocity than normal flow (Bedient 2008). Data collected from this research has shown that infiltration observed during sheet flow across a pavement is lower than infiltration rates measured by ASTM C1701, on the same pavement (ASTM C1701 2009). ASTM 1701 uses a constant head test to determine pervious concrete’s infiltration rate. The infiltration rates calculated by ASTM 1701 are shown in Chapter 3. ASTM C1701 calculates infiltration based on standing water on the pavement, while the model calculates infiltration based on sheet flowing water across the pavement. The majority of pervious concrete pavements are specified based on its infiltration rate, and is expected to meet that rate when it’s installed.

The relationship between the ASTM 1701 infiltration rate and the MCFBP (Capture Discharge) is shown in Figure 33. Figure 33 provides a visual on the relationship between the ASTM 1701 infiltration rate and the MCBP, the capture discharge. Figure 33 shows two trends, first that as the slope increases the MCFBP decreases, and this is as
expected. The second trend in Figure 33 is that the decrease in MCFBP between cross slopes is greater at higher infiltration rates. All infiltration rates have been analyzed based on the standard ASTM 1701, which calculates infiltration with a 0% cross slope.

![Figure 33 Plot of the ASTM 1701 infiltration verse the MCBP(Capture Discharge).](image)

Statistical analysis of the data represented in Figure 33 indicated that over the entire range of data there is no statistical difference between the regression slope for the 0% slope line and the regression slope for the 10% line. Additional analysis of the data from the lower three void contents shows that there is a statistical difference in the regression slope of the 0% slope line and the 10% slope line (Blank 1980). The lack of statistical difference over the range of data is believed to be due to the difference in orders of magnitude from the capture discharge to the ASTM C1701 infiltration.

Another way to examine infiltration rates is to look at the actual infiltration rates at different cross slopes of the pavement, with the use of sheet flowing water instead of the
constant head method utilized by ASTM 1701. The infiltration rate observed by the model was calculated from the effective intensities. To convert effective intensities into infiltration rate, a simple unit transformation was performed. Effective intensities are in cfs and infiltration rates need to be in inches per hour, thus, multiplying the effective intensities by 43,200 (3600 seconds/hour times 12 inches/foot) will produce infiltration rates. The calculated infiltration rates at by-pass are shown in Table 15. The infiltration rates shown in Table 15 are average values of three blocks, and are based off the MCFBP.

<table>
<thead>
<tr>
<th>Slope</th>
<th>45% Voids</th>
<th>35% Voids</th>
<th>25% Voids</th>
<th>20% Voids</th>
<th>15% Voids</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0%</td>
<td>3300</td>
<td>1590</td>
<td>770</td>
<td>520</td>
<td>330</td>
</tr>
<tr>
<td>0.5%</td>
<td>3100</td>
<td>1470</td>
<td>630</td>
<td>400</td>
<td>250</td>
</tr>
<tr>
<td>1.0%</td>
<td>3000</td>
<td>1510</td>
<td>580</td>
<td>390</td>
<td>240</td>
</tr>
<tr>
<td>2.0%</td>
<td>2900</td>
<td>1340</td>
<td>600</td>
<td>390</td>
<td>210</td>
</tr>
<tr>
<td>5.0%</td>
<td>2700</td>
<td>1220</td>
<td>580</td>
<td>330</td>
<td>220</td>
</tr>
<tr>
<td>10.0%</td>
<td>2500</td>
<td>1040</td>
<td>530</td>
<td>220</td>
<td>170</td>
</tr>
</tbody>
</table>

Tables 15 shows a relationship between slope and infiltration. As the slope is increased the infiltration rate decreases. The infiltration rates of the different void contents will decrease similarly to the MCF’s shown in Figure 33, due to the direct relationship of the infiltration and the MCF.

Figure 34 shows the actual infiltration verse infiltration determined by ASTM C1701, for the same high void sample at a horizontal slope (ASTM 2009). For very low
infiltration values the difference is small, but as the run-on velocity increases there is a substantial reduction in the infiltration capacity.

Figure 34 Actual infiltration verse ASTM C1701 infiltration.
RESULTS AND DISCUSSION (CLOGGED)

In regions that have cold climates, it is crucial for roadways to be treated for icy and slick conditions during inclement weather. To reduce the buildup off ice on roadways and provide traction for motorists a combination of deicers and sand is generally used. While the deicers themselves are generally not a problem with regards to clogging of pervious concrete, the deicers is either liquid or a substance able to dissolve. The sand used in deicing could infiltrate and clog the pervious concrete used in highway shoulders. This chapter will discuss how the model was used to mimic de-icing treatments and the results generated.

The amount of sand used in the model was calculated based off a few general assumptions. The first assumption made was that the design would be for a two lane, 12 feet per lane, highway. The second assumption was a spread rate of 600 pounds per lane per mile of de-icing mixture. And the third assumption was a deicing mixture ratio of 1:1 salt to sand. These assumptions were based off of amounts used in Massachusetts and Wisconsin (Massachusetts 2006, Walker 2005).

In the model sand was initially spread upstream of the pervious concrete blocks and allowed to be suspended in the water and carried into the sample. This method is shown in Figure 35. Due to the techniques used to secure the pervious concrete blocks in the model
the sand would become trapped before entering the concrete. The entrapped sand can be seen in Figure 36. To mitigate this problem the sand was windrowed on the upstream most part of the pervious concrete blocks, this is shown in Figure 37. Figure 38 shows how the sand clogged the pervious concrete blocks. The larger pieces of sand are more visible on the top layer while the smaller pieces of sand infiltrated deeper into the block.

![Figure 35 Sand spread upstream of pervious concrete block.](image)

Figure 35 Sand spread upstream of pervious concrete block.
Figure 36 Entrapped sand upstream of pervious concrete block

Figure 37 Sand windrowed at upstream-most part of pervious concrete block
Figure 38 Sand clogging the pervious concrete block.

The hydraulic flume was modified for the clogging tests and was fixed at a 2% cross slope to simulate a roadway. Initial clogging testing involved recording data after each dosage rate to identify if there is a trend. The MCF was calculated the same way as described in Chapter 5. Figures 39 and 40 show the effects of deicers, per application, have on the MCF. Figure 39 is data for one of the 25% void content blocks. Figure 40 is data for one of the 20% void content blocks.
Figure 39 Plot of applications of sand verse MCF for 25% void content block.

Figure 40 Plot of applications of sand verse MCF for the 20% void content block.
The trendlines shown in Figures 39 and 40, show that as sand is applied to the samples the MCF begins to decrease. The trendlines also show a tendency of the samples to stabilize towards the higher number of applications of sand. This is due to the amount of sand in the samples, towards the higher number of applications of sand it was observed that more of the sand began to by-pass the samples and not infiltrate or be caught in the surface.

Due to the tendency of the MCF to stabilize, data was only recorded for the every fifth application of sand. The applications of sand remained consistent and were applied one at a time, and allowed to infiltrate the samples. The data recorded for the application of sand of the samples is shown in Tables 16 through 20.

Table 16 MCF after Application of Sand for the 45% Void Content Samples

<table>
<thead>
<tr>
<th>Application of Sand</th>
<th>MCFBBP, cfs</th>
<th>Effective Intensity, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0933</td>
<td>0.0676</td>
</tr>
<tr>
<td>5</td>
<td>0.0713</td>
<td>0.0517</td>
</tr>
<tr>
<td>10</td>
<td>0.0599</td>
<td>0.0434</td>
</tr>
<tr>
<td>15</td>
<td>0.0496</td>
<td>0.0359</td>
</tr>
<tr>
<td>20</td>
<td>0.0342</td>
<td>0.0248</td>
</tr>
</tbody>
</table>
Table 17 MCF after Application of Sand for the 35% Void Content Samples

<table>
<thead>
<tr>
<th>Application of Sand</th>
<th>MCFBP, cfs</th>
<th>Effective Intensity, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0427</td>
<td>0.0309</td>
</tr>
<tr>
<td>5</td>
<td>0.0288</td>
<td>0.0209</td>
</tr>
<tr>
<td>10</td>
<td>0.0193</td>
<td>0.0140</td>
</tr>
<tr>
<td>15</td>
<td>0.0119</td>
<td>0.0086</td>
</tr>
<tr>
<td>20</td>
<td>0.0075</td>
<td>0.0054</td>
</tr>
</tbody>
</table>

Table 18 MCF after Application of Sand for the 25% Void Content Samples

<table>
<thead>
<tr>
<th>Application of Sand</th>
<th>MCFBP, cfs</th>
<th>Effective Intensity, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0190</td>
<td>0.0138</td>
</tr>
<tr>
<td>5</td>
<td>0.0099</td>
<td>0.0071</td>
</tr>
<tr>
<td>10</td>
<td>0.0056</td>
<td>0.0041</td>
</tr>
<tr>
<td>15</td>
<td>0.0035</td>
<td>0.0025</td>
</tr>
<tr>
<td>20</td>
<td>0.0028</td>
<td>0.0020</td>
</tr>
</tbody>
</table>

Table 19 MCF after Application of Sand for the 20% Void Content Samples

<table>
<thead>
<tr>
<th>Application of Sand</th>
<th>MCFBP, cfs</th>
<th>Effective Intensity, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.012</td>
<td>0.0087</td>
</tr>
<tr>
<td>5</td>
<td>0.00463</td>
<td>0.0034</td>
</tr>
<tr>
<td>10</td>
<td>0.00283</td>
<td>0.0021</td>
</tr>
<tr>
<td>15</td>
<td>0.00175</td>
<td>0.0013</td>
</tr>
<tr>
<td>20</td>
<td>0.0012</td>
<td>0.0009</td>
</tr>
</tbody>
</table>
Table 20 MCF after Application of Sand for the 15% Void Content Samples

<table>
<thead>
<tr>
<th>Application of Sand</th>
<th>MCFBP, cfs</th>
<th>Effective Intensity, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0067</td>
<td>0.0048</td>
</tr>
<tr>
<td>5</td>
<td>0.0027</td>
<td>0.0020</td>
</tr>
<tr>
<td>10</td>
<td>0.0015</td>
<td>0.0011</td>
</tr>
<tr>
<td>15</td>
<td>0.0008</td>
<td>0.0006</td>
</tr>
<tr>
<td>20</td>
<td>0.0006</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

The data shown in Tables 16 through 20 indicate that as the more sand is applied to the sample the lower the MCF, as expected. The results indicates that as the void content of the samples decreases so does the final MCFBP. The results also indicate that there is a greater difference in the initial applications of sand than in the later applications. The decrease in change between the later applications of sand is due to the void content being reduced as the sand infiltrates and clogs the sample, forcing more of the sand to by-pass, and creating a smaller change in the amount of clogged space in the sample. Figure 41 shows the effects of applications of sand on the ASTM C1701 infiltration rate.
The relationship between infiltration rate and capture discharge shown in Figure 41 behaves as expected from the previous data presented. The capture discharge is shown to decrease with the additional applications of sand. This decrease in capture discharge is more substantial as the infiltration rate increases.

Additional Testing

After the final application of sand to the pervious concrete blocks three additional tests were performed. The tests were performed on one block each, therefore the data presented in this section is only representative of one block.

The first additional test was to determine if any of the sand that infiltrated the block would become mobile once the block was dried. To analyze this, the samples were allowed
to completely dry after the final application of sand was added. Once the blocks were completely dry they were tested again to determine MCFBP. The effects on drying of the sample on the MCF are shown in Table 21.

Table 21 Effect of Drying on MCF

<table>
<thead>
<tr>
<th>Void Content</th>
<th>After Sand Application</th>
<th>After Drying</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Original</td>
<td>Clogged</td>
</tr>
<tr>
<td>45%</td>
<td>0.083</td>
<td>0.0218</td>
</tr>
<tr>
<td>35%</td>
<td>0.034</td>
<td>0.0084</td>
</tr>
<tr>
<td>15%</td>
<td>0.007</td>
<td>0.0007</td>
</tr>
</tbody>
</table>

*MCF Values are in cfs

While the MCF did not return to the original MCF before application of sand, the data suggests that is allowed to dry the MCF will increase slightly. Further investigation would be needed to determine the effects of the sample drying in between each application of sand.

The second additional test performed was to determine the effects of vacuuming the pervious concrete blocks on the MCF. Typical maintenance of pervious concrete pavements includes vacuuming to remove debris clogging the pavement. To analyze the effects of vacuuming the pervious concrete blocks were allowed to completely dry. Once dry the blocks were vacuumed using a standard shop vacuum with 6.5 peak horsepower. The effects of vacuuming are shown in Table 22.
The effects of vacuuming on the pervious concrete blocks shown in Table 22 indicate similar results to the effects of drying. The results indicate that vacuuming will increase the MCF. The vacuuming results are also similar to the drying results in that neither resulted in complete mitigation of the loss of MCF due to clogging. To analyze which remediation process is more effect the percent loss from before clogging to after remediation is shown in Table 23. The values shown are the percentage increase of the MCF due to remediation from the clogged MCF.

Table 22 Effect of Vacuuming on MCF

<table>
<thead>
<tr>
<th>Void Content</th>
<th>After Sand Application</th>
<th>After Vacuuming</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Original</td>
<td>Clogged</td>
</tr>
<tr>
<td>45%</td>
<td>0.104</td>
<td>0.0466</td>
</tr>
<tr>
<td>35%</td>
<td>0.049</td>
<td>0.0075</td>
</tr>
<tr>
<td>25%</td>
<td>0.016</td>
<td>0.0025</td>
</tr>
<tr>
<td>20%</td>
<td>0.014</td>
<td>0.0011</td>
</tr>
<tr>
<td>15%</td>
<td>0.007</td>
<td>0.0007</td>
</tr>
</tbody>
</table>

*MCF Values are in cfs

Table 23 Comparison of Drying and Vacuuming for Remediating Clogging

<table>
<thead>
<tr>
<th>Void Content</th>
<th>% Increase in MCF from Vacuuming</th>
<th>% Increase in MCF from Drying</th>
</tr>
</thead>
<tbody>
<tr>
<td>45%</td>
<td>0%</td>
<td>2%</td>
</tr>
<tr>
<td>35%</td>
<td>260%</td>
<td>30%</td>
</tr>
<tr>
<td>25%</td>
<td>350%</td>
<td>-</td>
</tr>
<tr>
<td>20%</td>
<td>520%</td>
<td>-</td>
</tr>
<tr>
<td>15%</td>
<td>510%</td>
<td>260%</td>
</tr>
</tbody>
</table>
The results shown in Table 23 show a significant increase in MCF due to remediation for all void contents except the 45%. The effects on the 45% void content sample are minimal due to the amount of voids. The more voids associated with the 45% void content sample enabled the sand to infiltrate deeper into the block which made remediation more difficult.

Table 23 also indicates that as the void content decreases the effects remediation increase. The increase in effectiveness of remediation is due to the diminishing of voids. With lower void amounts the sand was not able to infiltrate as deep, leaving a majority of the sand near the top, making remediation more efficient.

All remediation techniques discussed in this section were implemented after the completion of the 20th application of sand. Further investigation of remediation earlier in the stages of clogging is still needed. But, it is believed that these results will be more substantial if performed prior to complete clogging of the pervious concrete.
CHAPTER 7
HYDRAULIC DESIGN OF PERVIOUS CONCRETE

The laboratory flow rates and discharges were converted into effective rainfall intensity to correlate the data with design storm frequencies. Rainfall intensity is based on the duration of rainfall, and the area affected by rainfall (Bedient 2008). The effective intensities shown in Figure 42 are based on the area of the concrete sample and are shown as capture discharge (amount of flow that will infiltrate the pavement in cubic feet per second, per unit width, per unit length). The capture discharge is plotted against the infiltration rate in Figure 42. Figure 42 shows the linear relationship between the infiltration rate and the capture discharge. Figure 42 also shows that at increased cross slopes the relationship between capture discharge and design infiltration remains linear, but the amount of capture discharge is lower. This is an important relationship because it verifies the assumption that as slope increases the amount of infiltration decreases.

For ease of design Figure 43 was generated from Figure 42 to show the capture discharges for varying slopes at lower infiltration rates.
Figure 42 Capture discharge verse pavement infiltration rate.

Figure 43 Capture discharge verse pavement infiltration rate, at lower infiltration rates.
The potential variables involved in permeable pavement shoulder design are: pavement infiltration capacity, pavement cross slope, lane widths, shoulder widths, and design storm frequency. American Public Works Association, (APWA), 5600 was used to determine the unit discharges for different storm frequencies (American 2010). A time to concentration was assumed to be five minutes. Since storm frequencies are a function of basin area, the width of the roadway was used to calculate unit discharges for different storm frequencies. Figure 44 shows the calculated discharges that correspond to different design storm frequencies and roadway widths for the Kansas City metropolitan area. Adaptation of this design methodology would only require calculating rainfall intensities based on the region of interest.

Figure 44 Unit discharge versus roadway width based on storm frequencies
To account for the effects of clogging on pervious concrete shoulder the Clogged MCF was calculated (Calculation shown in Chapter 6). To produce a realistic design aid the 45% and 35% void data was omitted, because of their low strengths, and a linear trendline of the remaining data was produced. Figure 45 shows the plot of the clogged MCF for varying void contents.

Figure 45 Clogged maximum capture flowrate for varying void contents.

Figure 45 can be used in connection with the other design aids in this chapter to determine the required void content given a captured discharge, or to determine the capture discharge of a given void content.

Figure 30 contains the relationships between the ASTM C1701 infiltration rates and the actual infiltration rates. This figure can be used to determine actual field infiltration...
rates given the design infiltration rates from ASTM C1701. Figure 34 can also be used to determine ASTM C1701 infiltration rates given the required field infiltration rates.

Hydraulic Design Example

This example will determine the minimum infiltration rate of a pervious concrete shoulder for management of a ten year storm event. The design example includes one half of a typical highway section including two 12 foot lanes (3.7 m) and a 12 foot (3.7 m) pervious concrete shoulder at a 2% cross slope. The width of this highway centerline to edge of shoulder is 36 feet (11 m).

The first step in this design process is to determine the unit discharge needed to be captured by the pervious concrete highway shoulder for the given storm event. Figure 46 indicates how Figure 44 is used to determine the unit discharge for the design example, given 36 feet center line to edge of shoulder and the 10 year storm event.
Step one determined that the required discharge the shoulder would need to capture is $7.3 \times 10^{-2}$ cfs. Step two requires using this data to determine the unit discharge needed for the shoulder to capture. The unit discharge is simply the discharge found in step one divided by the width of the highway shoulder, in this example 12 feet. Therefore the unit discharge required for capture is equal to $7.3 \times 10^{-2}$ cfs (From step 1) divided by 12 feet (The width of the shoulder), which generates a value of $0.608 \times 10^{-2}$ cfs/unit.

Step three is to use the unit discharge calculated in step two to determine the ASTM C1701 design infiltration rate. Figure 47 indicates how Figure 43 can be used to determine the ASTM C1701 design infiltration rate given the unit discharge required.
Figure 47 Step three of the design example.

Figure 47 indicates that an infiltration rate of about 410 in/hr is required for this design, to account for error and for ease of design this is rounded up to 500 in/hr.

The previously explained example does not account for clogging of the pervious pavement. To account for clogging an alternative step three is needed. Figure 48 shows how Figure 41 is used to determine required infiltration rate. This alternative step three uses the capture discharge of 0.608 cfs obtained in step two, and shows the design for a pavement that will undergo 10 applications of sand. It should be noted that due to the scope of research, Figure 48 is only representative of a sample pavement at a 2% cross slope.
From alternative step three, Figure 48, it can be concluded that in order to account for clogging effects associated with 10 applications of sand, a pavement with an infiltration rate of around 900 in/hr would be required. In addition to designing for the effects of clogging for the application of sand in de-icing mixtures, preventative measures could be made to reduce the potential of clogging, such as using alternative de-icing mixtures.

Figure 49 is provided as a quick reference for the design of pervious concrete highway shoulders.
Figure 49 Quick reference design example of a pervious concrete highway shoulder.

**Step 1:** Using the determined width from center line to edge of shoulder, and the design storm frequency, determine the unit discharge from Figure 6. For this example the width is 36 ft (11 m) and the design storm frequency is the ten year storm. Figure 6 shows a unit discharge of approximately $7.3 \times 10^{-2}$ cfs ($0.2 \times 10^{-2}$ m$^3$/s).

**Step 2:** Using the unit discharge determined in step 1 determine the required capture discharge (unit discharge divided by the width of the shoulder). The unit discharge determined in step one is $7.3 \times 10^{-2}$ cfs ($0.2 \times 10^{-2}$ m$^3$/s) and the width of the shoulder was given as 12 feet (3.7 m). This produces a required capture discharge of $0.608 \times 10^{-2}$ cfs ($0.054 \times 10^{-2}$ m$^3$/s) per unit width per unit length.

**Step 3:** Using the capture discharge determined in step 2, and Figure 5, determine the minimum ASTM 1701 infiltration rate required to capture the rainfall from this storm event. From Figure 5 a pavement with an infiltration rate greater than 400 in/hr (1000 cm/hr) will capture rainfall from a ten year storm for the given highway parameters.
CHAPTER 8
CONCLUSIONS

Stormwater reduction and filtration are necessities as impervious surfaces continue to be constructed. As regulations and enforcements for stormwater management continues to increase so must the design and effectiveness of the stormwater BMPs. Pervious concrete is a viable option to capture and filter stormwater runoff from impervious surfaces. BMPs to fully utilize pervious concrete as a solution to stormwater runoff need more attention, especially for hydraulic design.

The hydraulic response of the pervious concrete showed to have an inverse relationship with the cross slope of the pavement. As the cross slope increased the maximum capture flowrate before by-pass began significantly decreased. This is most likely due to the velocities associated with the sheet flowing water at higher cross slopes. The higher velocities associated with the higher cross slopes prevented the water to fully infiltrate the permeable pavement and the by-pass flow of water increased. The effects of decreased capture flowrate were more pronounced for lower void contents. The research also showed that the permeable pavements reached a point of consistency, where the infiltration became independent of flow.

The infiltration rates calculated from the model showed to be significantly lower then the infiltration rates calculated using the method described in ASTM C1701. The ASTM C1701 infiltration rates are determined using a constant head test were the water is
allowed to infiltrate the pavement solely through a vertical manner. The infiltration rates in the model were subjected to sheet flowing water, the water was travelling horizontally across the pavement and had to transition into flowing vertically through the pavement.

The hydraulic response of clogging of the permeable pavement due to sand used in de-icing of roadways followed similar trends as the unclogged permeable pavements. With the higher additions of sand having more pronounced effects on the pavements. Similarly to the trend in the unclogged pavements, the clogged pavements reached a point of consistency. This point of consistency occurred when the sand could no longer infiltrate the samples, and began to by-pass them all together. This effect led to the consistency in infiltration of the samples.

Two remediation techniques were analyzed in this research, the effects of vacuum cleaning the permeable pavements, and the effects of drying of the permeable pavements. The results were positive for both techniques, and showed an increase in infiltration after the remediation. The results showed that for lower void contents the remediation was more pronounced, with the vacuum cleaning of the permeable pavements having a greater effect then the drying. While the remediation techniques improved the hydraulic response of the pavements, they did not return to their full design capacity. Remediation was performed after twenty applications of sand. Further investigation of the effects of clogging remediation, at earlier stages, would be more effective in restoring the pavements to their design capacities.
This research has shown that the captured discharge, infiltration, is a function of slope and pavement permeability. It is the recommendation of the researcher that to fully utilize pervious pavements, design considerations of slope are a necessity. A design methodology for permeable highway shoulders has been included in this thesis. The design methodology accounts for various storm events and pavement cross slopes to develop a recommendation for an ASTM C1701 infiltration rate, to be used in the pavement design. The design methodology shows that permeable pavements can be used as effective BMPs if the proper design considerations are taken.

Future Research and other Applications

Additional research would be beneficial in creating a full design methodology, which would increase the effectiveness of permeable pavements as BMPs. Investigations into additional slopes and void contents would be beneficial. While it is assumed that the hydraulic response of clogged permeable pavements at cross slopes other than 2% would mimic those of the 2% cross slope, additional research of clogged permeable pavements at additional slopes would solidify the assumption.

Research into the use of permeable pavements for roadway shoulders as BMPs in applications other than highway design is needed. Permeable pavements could be used in curb in gutter systems if the right design is created.

Prototype models, such as this one, should be designed and tested to investigate the hydraulic effects of other types of clogging. The research presented in this thesis solely investigated the effects of filtered water, and clogging due to sand in permeable pavements. While filtered water is a viable representation of stormwater, the reality is that
some water encountered in real life situations will be unfiltered. The water entering the permeable pavements can contain other debris that will clog the pavement in different manners.
REFERENCES


APPENDIX A

PERFORMANCE CURVES

A 1 Performance Curve for the 45 % Void Content for block 1 at 0% slope.

A 2 Performance Curve for the 45 % Void Content for block 1 at 0.5% slope.
A 3 Performance Curve for the 45 % Void Content for block 1 at 1% slope.

A 4 Performance Curve for the 45 % Void Content for block 1 at 2% slope.
A 5 Performance Curve for the 45 % Void Content for block 1 at 5% slope.

A 6 Performance Curve for the 45 % Void Content for block at 10% slope.
A 7 Performance Curve for the 45 % Void Content for block 2 at 0% slope.

A 8 Performance Curve for the 45 % Void Content for block 2 at 0.5% slope.
A 9 Performance Curve for the 45 % Void Content for block 2 at 1% slope.

A 10 Performance Curve for the 45 % Void Content for block 2 at 2% slope.
A 11 Performance Curve for the 45 % Void Content for block 2 at 5% slope.

A 12 Performance Curve for the 45 % Void Content for block 2 at 10% slope.
A 13 Performance Curve for the 45 % Void Content for block 3 at 0% slope.

A 14 Performance Curve for the 45 % Void Content for block 3 at 0.5% slope.
A 15 Performance Curve for the 45 % Void Content for block 3 at 1% slope.

A 16 Performance Curve for the 45 % Void Content for block 3 at 2% slope.
A 17 Performance Curve for the 45 % Void Content for block 3 at 5% slope.

A 18 Performance Curve for the 45 % Void Content for block 3 at 10% slope.
A 19 Performance Curve for the 35 % Void Content for block 1 at 0% slope.

A 20 Performance Curve for the 35 % Void Content for block 1 at 0.5% slope.
A 21 Performance Curve for the 35 % Void Content for block 1 at 1% slope.

A 22 Performance Curve for the 35 % Void Content for block 1 at 2% slope.
A 23 Performance Curve for the 35 % Void Content for block 1 at 5% slope.

A 24 Performance Curve for the 35 % Void Content for block 1 at 10% slope.
A 25 Performance Curve for the 35 % Void Content for block 2 at 0% slope.

A 26 Performance Curve for the 35 % Void Content for block 2 at 0.5% slope.
A 27 Performance Curve for the 35 % Void Content for block 2 at 1% slope.

A 28 Performance Curve for the 35 % Void Content for block 2 at 2% slope.
A 29 Performance Curve for the 35 % Void Content for block 2 at 5% slope.

A 30 Performance Curve for the 35 % Void Content for block 2 at 10% slope.
A 31 Performance Curve for the 35 % Void Content for block 3 at 0% slope.

A 32 Performance Curve for the 35 % Void Content for block 3 at 0.5% slope.
A 33 Performance Curve for the 35 % Void Content for block 3 at 1% slope.

A 34 Performance Curve for the 35 % Void Content for block 3 at 2% slope.
A 35 Performance Curve for the 35 % Void Content for block 3 at 5% slope.

A 36 Performance Curve for the 35 % Void Content for block 3 at 10% slope.
A 37 Performance Curve for the 20 % Void Content for block 1 at 0% slope.

A 38 Performance Curve for the 20 % Void Content for block 1 at 0.5% slope.
A 39 Performance Curve for the 20 % Void Content for block 1 at 1% slope.

A 40 Performance Curve for the 20 % Void Content for block 1 at 2% slope.
A 41 Performance Curve for the 20 % Void Content for block 1 at 5% slope.

A 42 Performance Curve for the 20 % Void Content for block 1 at 10% slope.
A 43 Performance Curve for the 20 % Void Content for block 2 at 0% slope.

A 44 Performance Curve for the 20 % Void Content for block 2 at 0.5% slope.
A 45 Performance Curve for the 20 % Void Content for block 2 at 1% slope.

A 46 Performance Curve for the 20 % Void Content for block 2 at 2% slope.
A 47 Performance Curve for the 20 % Void Content for block 2 at 5% slope.

A 48 Performance Curve for the 20 % Void Content for block 2 at 10% slope.
A 49 Performance Curve for the 20 % Void Content for block 3 at 0% slope.

A 50 Performance Curve for the 20 % Void Content for block 3 at 0.5% slope.
A 51 Performance Curve for the 20 % Void Content for block 3 at 1% slope.

A 52 Performance Curve for the 20 % Void Content for block 3 at 2% slope.
A 53 Performance Curve for the 20 % Void Content for block 3 at 5% slope.

A 54 Performance Curve for the 20 % Void Content for block 3 at 10% slope.
A 55 Performance Curve for the 25 % Void Content for block 1 at 0% slope.

A 56 Performance Curve for the 25 % Void Content for block 1 at 0.5% slope.
A 57 Performance Curve for the 25 % Void Content for block 1 at 1% slope.

A 58 Performance Curve for the 25 % Void Content for block 1 at 2% slope.
A 59 Performance Curve for the 25 % Void Content for block 1 at 5% slope.

A 60 Performance Curve for the 25 % Void Content for block 1 at 10% slope.
A 61 Performance Curve for the 25 % Void Content for block 2 at 0% slope.

A 62 Performance Curve for the 25 % Void Content for block 2 at 0.5% slope.
A 63 Performance Curve for the 25 % Void Content for block 2 at 1% slope.

A 64 Performance Curve for the 25 % Void Content for block 2 at 2% slope.
A 65 Performance Curve for the 25 % Void Content for block 2 at 5% slope.

A 66 Performance Curve for the 25 % Void Content for block 2 at 10% slope.
A 67 Performance Curve for the 25 % Void Content for block 3 at 0% slope.

A 68 Performance Curve for the 25 % Void Content for block 3 at 0.5% slope.
A 69 Performance Curve for the 25 % Void Content for block 3 at 1% slope.

A 70 Performance Curve for the 25 % Void Content for block 3 at 2% slope.
A 71 Performance Curve for the 25 % Void Content for block 3 at 5% slope.

A 72 Performance Curve for the 25 % Void Content for block 3 at 10% slope.
A 73 Performance Curve for the 15 % Void Content for block 1 at 0% slope.

A 74 Performance Curve for the 15 % Void Content for block 1 at 0.5% slope.
A 75 Performance Curve for the 15 % Void Content for block 1 at 1% slope.

A 76 Performance Curve for the 15 % Void Content for block 1 at 2% slope.
A 77 Performance Curve for the 15 % Void Content for block 1 at 5% slope.

A 78 Performance Curve for the 15 % Void Content for block 1 at 10% slope.
A 79 Performance Curve for the 15 % Void Content for block 2 at 0% slope.

A 80 Performance Curve for the 15 % Void Content for block 2 at 0.5% slope.
A 81 Performance Curve for the 15 % Void Content for block 2 at 1% slope.

A 82 Performance Curve for the 15 % Void Content for block 2 at 2% slope.
A 83 Performance Curve for the 15 % Void Content for block 2 at 5% slope.

A 84 Performance Curve for the 15 % Void Content for block 2 at 10% slope.
A 85 Performance Curve for the 15 % Void Content for block 3 at 0% slope.

A 86 Performance Curve for the 15 % Void Content for block 3 at 0.5% slope.
A 87 Performance Curve for the 15 % Void Content for block 3 at 1% slope.

A 88 Performance Curve for the 15 % Void Content for block 3 at 2% slope.
A 89 Performance Curve for the 15 % Void Content for block 3 at 5% slope.

A 90 Performance Curve for the 15 % Void Content for block 3 at 10% slope.
VITA

Nathan Andrew Grahl was born on April 15th 1988 in Independence Missouri. He attended Archbishop O’Hara High School in Kansas City Missouri. He then studied pre-Engineering at Northwest Missouri State University before completing his Bachelors of Science in Civil Engineering from the University of Missouri- Kansas City. While at the University of Missouri- Kansas City Nathan became active in student organizations and became an officer in the following student organizations: Chi epsilon, Design Build Institute of America, and the American Concrete Institute. Nathan worked for several years in the labs at UMKC performing research for Dr. John Kevern and currently works for Black & Veatch in the Water Division at the office in Kansas City Missouri.